ROAD ENGINEERING ASSOCIATION OF ASIA & AUSTRALASIA

JOURNAL

PP 7021/8/93

No.8

ROAD ENGINEERING
ASSOCIATION OF
ASIA &
AUSTRALASIA

Journal

July 1996

JOURNAL

Publisher
- THE ROAD ENGINEERING ASSOCIATION OF ASIA & AUSTRALASIA
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P.J. Industrial Park, Jalan Kemajuan
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Selangor, Malaysia

Tel: 60-3-7571159
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Printer
- Percetakan Bahagia
3 Jalan Jejaka Dua, Taman Maluri
Kuala Lumpur

Lay-Out
- AC Designers Sdn. Bhd. Tel: 7808354

Cover:
Photo shows part of the highway network linking Pokhara to Batuwal in Northern Nepal.

BRIDGE MANAGEMENT SYSTEM

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Abstract

Currently there are about 4000 bridges along the Federal routes of Peninsular Malaysia. In order to ensure that every bridge is in serviceable condition, systematic maintenance strategy has to be adopted. Towards meeting this objective JKR has developed a Bridge Management System known as JKR-BMS to assist in the daily operation and management of bridges. The paper will provide an overview of JKR-BMS, its structure, current development and future development work that is needed for it to serve JKR better on its road policy and decision making process. In a nutshell the real value of the JKR-BMS is in its potential to consider all the bridge stock when allocating resources and establishing its maintenance policies.

Introduction

JKR started undertaking an in-house development of a bridge management system in 1988, with a system development team comprising of engineers of the Bridge Unit. A prototype BMS, which was later known as JKR-BMS was completed in 1991 with the database obtained from the National Axle Load Study which was carried out between 1986 and 1990. Under the funding of the World Bank, the JKR-BMS was later reviewed by two BMS experts from the USA. The JKR-BMS has since then been implemented in the Bridge Unit of JKR.

This paper will describe the JKR-BMS and highlights its current usage in JKR, and discuss the future directions for its implementation.

Bridge Management System Objective

Bridge managers must make the best use of the limited funds, balancing the need for safety, preservation of investment and serving the commerce and motoring public. They must monitor potentially severe safety problems such as bridges subject to collapse due to structural failure, scouring or lack of structural support. They must identify premature deterioration and damage bridges and use the funds to avoid further deterioration and correct damage through proper maintenance and repair. They also must strive to reduce inconvenience and disruptions to commerce and passenger travel, due to load and clearance restrictions or bridge closures.

Because of the magnitude and complexity of the bridge management problem, there is a need to establish a system to assist bridge managers to have quick access to bridge information, assess the conditions and allocate priorities and determine the best possible allocation of funds among various types of bridge maintenance, repair, rehabilitation, improvement and replacement works. Such a system is referred to as a Bridge Management System (BMS).

The Context of Bridge Management System

The BMS function has to be looked into a wider context beyond the procedures for analysing bridge data for the purpose of predicting maintenance and improvement needs, determining optimal policies, and recommending projects and schedules within budget and policy constraints. It is also not just a computerised database and decision support tools that supplies analyses and summaries of data, uses models and algorithms to make predictions and recommendations. It is as the name suggest, a system whereby various parties and processes are involved, and their functions and how they interact should be fully understood to make the system workable and fully utilised.

In this context a BMS should encompass the following:

(a) the clients or users, which determine the needs or requirements of the BMS

(b) the data which should be gathered and stored in the BMS

(c) the procedures, which is the "black box" in the system

(d) accessibility, the user interface and output

The clients and their needs can be represented by the context diagram as shown in Fig.1. Of course the main client is the Bridge Unit itself, where it's main objective is to plan and improve the development of the infrastructure and public services in the transportation system, specifically bridges, flyovers and culverts for roads, so that they are safe, of high quality and economical, to enable to fulfill the country's social and economic development. The BMS should thus be able to:

(a) maintain an inventory of all bridges under the jurisdiction of the relevant bridge authority

(b) provide specific information on any bridge on request

(c) provide a statistical analysis on various aspects of the bridge stock

(d) provide priority ranking and listing of bridges giving the appropriate improvement works to be undertaken

(e) provide cost estimates for the various types of improvement works, including economic analyses to prepare optimum budget allocations

(f) able to show the impacts on any alternative course of action

(g) provide information on the accessibility of vehicles

(h) present the information and output in format appropriate to the user request

(i) present itself with a very user friendly interface

The JKR-BMS

JKR-BMS provides significant improvement over the card system which was already in existence in JKR since the seventies and well outdated. Being computerised and designed to suit the present needs the system will no doubt be able to save valuable time and the money allocated for the repair and rehabilitation works will be well spent.

The system was developed to enable different levels of users to use it according to their need. Managers for instance, will be interested to know the ranking of bridges according to worthiness to take action first, because it will help him to decide on the necessary maintenance and rehabilitation strategies best suited for those set of bridges.

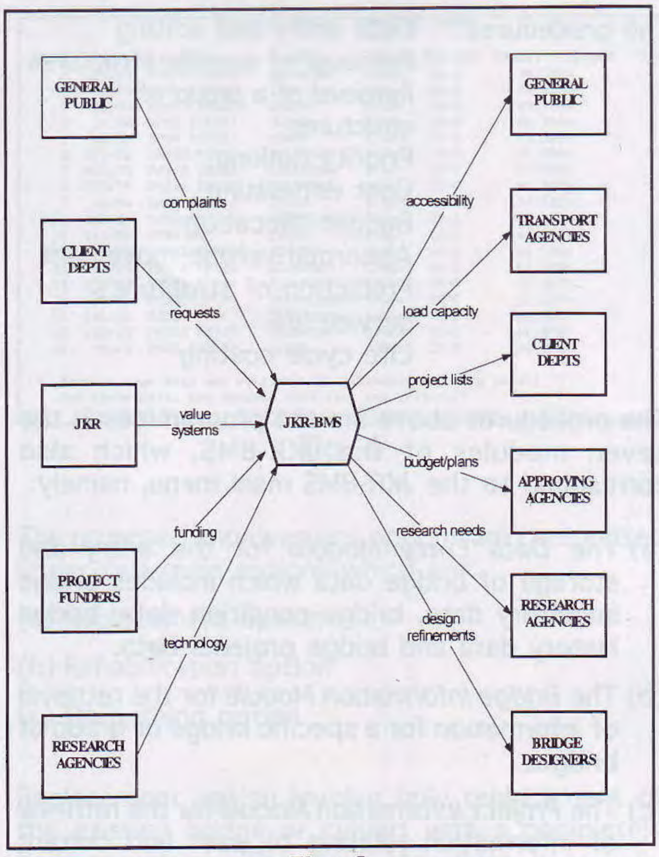

Fig. 1

In the context discussed above, the JKR-BMS can be seen to include the following:-

The Clients/Users:	Road Branch of the PWD Ministry of Works Government Departments and Agencies Consultants Transporters Research Agencies The Public
The database:	Identification and Locational data Geometrical and design data Construction and Historical data Condition data Capacity data Traffic data Services and utilities Cost data
User interface:	Data entry and editing Selection of procedures and information Information display and output Manuals and guides

The procedures: Data entry and editing
Retrieval of specific structures
Retrieval of a group of structures
Priority ranking
Cost estimating
Budget allocation
Abnormal vehicle movement
Prediction of structure's service life
Life cycle costing

The procedures above are the programmes in the seven modules of the JKR-BMS, which also correspond to the JKR-BMS main menu, namely:

(a) The *Data Entry Module* for the entry and storage of bridge data which includes bridge inventory data, bridge condition data, bridge history data and bridge projects data.

(b) The *Bridge Information Module* for the retrieval of information for a specific bridge or group of bridges.

(c) The *Project Information Module* for the retrieval of information relating to past and current projects.

(d) The *Bridge Priority Module* for priority ranking among all bridges or group of bridges for improvement and maintenance works according to a systematic and predetermined analytical process.

(e) The *Improvement Proposal Module* for life cycle costing and service life estimates for various improvement alternatives for individual bridges.

(f) The *Costing and Budget Module* for cost allocation for improvement works within the specified years and budget amount.

(g) *Abnormal Vehicle Movement Module* for indication of the capacity of the bridges along the chosen route for a specified abnormal vehicle.

Data Management

Data management involves the collection and storage of relevant bridge data, which can be categorised into the following:

(a) Bridge inventory data which includes data on bridge location, type, components and dimensions.

(b) Bridge condition data which describes the bridge in terms of its physical condition and carrying capacity.

(c) Bridge history data which summarises the projects that had been completed on the bridge.

Data entry is performed after two instances. The first is when a new bridge is completed, where all data is freshly input under the "As-built Report" menu option.

The "As-built Report" data includes all data in the three categories above. The "As-built Report" also allows any works or improvement on existing bridges to be recorded. The second instance is after the inspection of existing bridges, where the updated bridge condition data is entered together with any change to the inventory data in the "Inspection Report" menu option.

Information Retrieval

Bridge Information

This module allows details of a particular bridge to be viewed on a predesigned form, which can also be printed (see Fig. 2 and Fig. 3). It also allows for the selection of a group bridges based on criteria specified by the user, and a listing of the selected bridges is displayed in tabular form under headings specified by the user .

```
BRIDGE INFORMATION - SPECIFIC STRUCTURE (VIEW)

ROUTE NO. FT005 , STRUCTURE NO. 661/30            INVENTORY , PAGE 1/3

DISTRICT, STATE : MANJONG, PERAK

CROSSING    : River              NAME OF RIVER     : PERAK
SKEW ANGLE  : Right Deck         NAME OF STRUCTURE : IDRIS SHAH II

YEAR OF CONSTRUCTION : 1972

GEOMETRY -                           MATERIAL : Concrete
 NO OF SPANS            :    10      SYSTEM   : Others
 MAX SPAN LENGTH (m)    :  42.64
 TOTAL LENGTH    (m)    : 420.00
 CLEAR WIDTH     (m)    :   7.30     TYPE -
 TOTAL WIDTH     (m)    :   9.60      DECK       : P.C.Beam & R.C.Slab
                                      ABUTMENT   : Retaining Wall
                                      PIER       : Solid Wall
ENVIRONMENT : Severe                  FOUNDATION : Steel Pipe Pile

Prev/Next Screen : PgUp/PgDn     (P)rint Inventory     E(X)it Inventory
```

Fig. 2

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BRIDGE INFORMATION - SPECIFIC STRUCTURE (VIEW)

ROUTE NO. FT005 , STRUCTURE NO. 661/30            INVENTORY , PAGE 2/3

EQUIVALENT AGE   :  23 yrs      CAPACITY               : STAL
                                DISCOUNTED CAPACITY :  8.8 SV

CONDITION RATINGS -
 FOUNDATION     : 0.0           AV DAILY TRAFFIC (ADT) :   10341
 ABUTMENT       : 2.0           % HEAVY VEHICLE        :      24
 PIER           : 2.0           MODIFIED ADT           :   11582
 GIRDER         : 1.0
 DECK           : 1.0
 PARAPET        : 2.0           HORIZONTAL CLEARANCE    (m) :  Not Av.
 BEARING        : 0.0
 JOINT          : 3.0
 DRAINAGE       : 0.0           NAVIGIBILITY                :   Yes
 SURFACING      : 2.0           HIGHEST FLOOD CLEARANCE (m) :  Not Av.
 SLOPE          : 3.0

WEIGHTED CONDITION : 1.8        VERTICAL CLEARANCE      (m) :  Not App.

Prev/Next Screen : PgUp/PgDn     (P)rint Inventory     E(X)it Inventory
```

Fig. 3

Project Information

This allows for the retrieval of information relating to past and current projects. Information that are kept include the name of designer, name of contractor, date of tender and completion and project cost.

Prioritisation

Prioritisation refers to the selection of structures for which priority ranking is given for carrying out improvement works. Higher priority point will reflect the urgency of works on the particular structure in a network. The Priority module can make the priority selection at the national network level (i.e. the whole database), or for in a particular state, district or route network specified by the user.

Prioritisation is based on the following criteria:

(a) Weighted condition

(b) Discounted load capacity

(c) Clear deck width

(d) Vertical clearance or flood clearance

(e) Average daily traffic

For each bridge, the values of the attributes are combined in a mathematical formula to calculate priority points. The bridges with the highest priority points are considered to be the best candidates for improvement work. The formula to calculate priority includes a weighting factor and value system for each attribute. The weighting factor indicates the relative importance of each factor to the others and normalized to unity. The value system indicates the value of the attribute when the highest priority points are assigned and when no priority points are assigned. Some flexibility is allowed for the user to change the limiting conditions for each of the criterion above. An example of the priority ranking for structures on Route FT003 is shown in Fig. 4.

Improvement Proposal

Improvement proposal module allows the user to select the best remedial options from a list of proposed alternatives made available in the system to be carried out on each of the bridges in the list. This is usually the follow up of the bridge priority analysis. Each of the alternatives is evaluated on the basis of simple economic analysis, where discounted cost of the improvement work alternatives are computed and compared.

PRIORITISATION - ROUTE NO. FT003 Page 1/19

RANK	STRUC#	DISTRICT	STATE	CROSS	SYS-MAT	PRIORITY POINTS
1	666/60	PASIR PUTIH	KELANTAN	River	SG-C	70.926*
2	32/20	KOTA TINGGI	JOHOR	River	SG-C	70.633*
3	38/90	KOTA TINGGI	JOHOR	River	SG-C	69.133*
4	34/30	KOTA TINGGI	JOHOR	River	SG-S	66.767*
5	42/80	KOTA TINGGI	JOHOR	River	PC-C	65.462*
6	677/30	PASIR PUTIH	KELANTAN	River	SG-C	65.036*
7	672/70	PASIR PUTIH	KELANTAN	River	SG-C	64.977*
8	675/70	PASIR PUTIH	KELANTAN	River	SG-C	64.967*
9	43/90	KOTA TINGGI	JOHOR	River	SG-S	64.522*
10	418/00	KEMAMAN	TERENGGANU	River	SG-C	63.922*
11	141/80	MERSING	JOHOR	River	SG-C	63.139*
12	20/80	JOHOR BAHRU	JOHOR	River	SG-C	62.510*
13	685/00	PASIR PUTIH	KELANTAN	River	SG-C	62.428*
14	674/90	PASIR PUTIH	KELANTAN	River	SG-C	62.362*
15	649/70	BESUT	TERENGGANU	River	SG-C	62.269*
16	646/50	BESUT	TERENGGANU	River	SG-C	62.208*
17	683/10	PASIR PUTIH	KELANTAN	River	SG-C	62.059*
18	40/60	KOTA TINGGI	JOHOR	River	CT-C	62.010*

* denotes some data not available in database, priority points for these datas are assumed zero (ie. not critical)

Prev/Next Screen : PgUp/PgDn (P)rint (C)odes E(X)it

Fig. 4

The proposed improvement options are categorised under two main options which are:

(a) Replacement option

(b) Rehabilitation option

(c) Do-nothing option

Replacement option involve only replacement of the existing bridge or culvert with a completely new structure. The new structure is usually different from the old one, depending on the size. Several replacement options can be considered.

Rehabilitation option involves extensive repair including upgrading and restoration. The capacity of bridges after being rehabilitated are expected to be equal or better than before.

Do-nothing Options assumes that nothing is done to the structure except for normal routine maintenance such as cleaning, repainting, and minor repair works.

The output of bridge improvement analysis are tabulated (Fig. 5) to compare discounted present costs and initial costs.

RESULT OF PRIORTISATION ANALYSIS

ROUTE NUMBER: FT003 STRUCTURE NUMBER: 303/00

IMPROVEMENT ALTERNATIVES	DISCOUNTED PRESENT COSTS	INITIAL COSTS
ALTERNATIVE 1: REPLACEMENT	$439213	$336000
ALTERNATIVE 2: REHABILITATION	$104019	$79940
ALTERNATIVE 3: DO NOTHING	$42899	$0

Page up/Page down: PgUp/PgDn e(X)it

Fig. 5

Costing and Budgeting

This module is used to select structures together with its recommended work action and estimated cost that are within a given budget. Analysis can be done at the national level, state level or according to a specified route. Budget planning can be done for a specified number of years up to a maximum of seven years. The output is in the form of a listing the structures ranking in order of priority for action, nature of work and estimated cost (Fig. 6)

BUDGETING - STATE LEVEL(PAHANG)

RANK	ROUTE	STRUC#	DISTRICT	W_NATURE	EST_COST	Y_ACTION
1	FT006	30/50	RAUB	REHAB	127296.00	1996
2	FT008	101/20	LIPIS	REHAB	63684.00	1996
3	FT006	39/00	RAUB	REHAB	95380.80	1996
4	FT006	36/20	RAUB	M&R	444.80	1996
5	FT008	42/70	RAUB	REHAB	326592.00	1996
6	FT008	4/70	BENTONG	REHAB	60480.00	1996
7	FT008	41/90	RAUB	REHAB	289008.00	1996
8	FT008	39/10	RAUB	REHAB	71424.00	1997
9	FT008	3/50	BENTONG	REHAB	28896.00	1997
10	FT064	86/70	JERANTUT	M&R	1317.72	1997
11	FT008	62/70	RAUB	REHAB	835200.00	1997

notes : Cost are based on assumptions = unit rate x overall area
where unit rates are dependant on the suggested work nature

PgUp - Page Up , PgDn - Page Down , (P)rint , (S)ummary , E(X)it

Fig. 6

Abnormal Vehicle Movement

Abnormal Vehicles are special vehicles used to carry very heavy indivisible loads such as turbine weighing up to a few hundred tonnes. The Abnormal Vehicle module will compare the load effect due to abnormal vehicle with the load-carrying capacity of each of the bridges, in the designated route, to be crossed by the vehicle. The user enters the specified route including the beginning and ending points that the abnormal vehicle wishes to use. The user also has to specify the load and axle spacings of the vehicle (Fig. 7) The affected bridges can be listed out from the computer

PASSAGE OF PROPOSED ABNORMAL VEHICLE

ROUTE INVOLVED			VEHICLE CONFIGURATION			
Route	Origin	Destination	Axle Load	Axle Dist.	Axle Load	Axle Dist.
ft003	300.0	320.0	6.0	1.5		
			15.0	3.0		
			15.0	-		

Axle # 3 out of 3
PROCESSING DATA:IPlease wait....
Load per axle (TON) = 15.0
DistNumber of bridges ==83.0
Analysis completed = 3

Fig. 7

database. Those which are found to be under capacity will be highlighted.

Implementation of JKR-BMS

The JKR-BMS is currently being installed at the Bridge Unit of the JKR Headquarters. It can be accessed from the JKR network by users who have the proper access code. Data entry and editing is restricted to the BMS group of the Bridge Unit.

The JKR-BMS has served the Bridge Unit quite well as far as keeping a database and assisting in the selection and allocation of priority for bridge replacement and rehabilitation. However getting timely and accurate data is probably the most difficult aspect in keeping the system current and useful. Until mid-1995 most of the data kept in the JKR-BMS was obtained during the National Axle Load Study with few additional data for newly completed bridges. Then only about 2500 bridges of the total estimated number of about 4000 bridges on Federal Roads in Peninsular Malaysia were kept in JKR-BMS. Concerned about the condition of our bridges, JKR management decided to institute a mandatory annual bridge inspection which was carried out by staff from JKR District offices and JKR Special Units in early 1995, and the opportunity was taken to take stock of all bridges on the Federal Roads during the inspection. Almost 3700 bridges including main culverts on major Federal Roads were inspected, and all inspection reports were entered into JKR-BMS. Thus currently JKR-BMS keeps an inventory of these bridges, including the condition ratings after confirmatory inspection was carried out by experienced engineers from the Bridge Unit.

The inspection conducted thus far is superficial in nature and detailed inspection is needed in order to accurately ascertained the true conditions of individual bridge structures. The information will assist JKR to calibrate the remaining service life of the structures. Coupled with the age of the structures and the maintenance and repair work (cost) that the structure has undergone it is envisaged that the life cycle costing of the structure throughout its design life can be ascertained. The authors are of the opinion that this information is vital toward ensuring that our bridge stocks have the necessary safety factors against structural collapse due to overloading, deterioration and scouring effect and allocating priority for funding scheduled maintenance work.

In order to determine the structural capacity of bridges JKR has carried out and recently completed

a study and has established a methodology for assessment of structural capacity. Information obtained from the study for 203 bridges and further assessment on the remaining bridges will form an important input data to the JKR-BMS. Computer softwares called BRASS and RESIST were developed to assist the engineers in carrying out the assessment.

Further Development and Enhancement of JKR-BMS

With such a fast pace in the development of IT, the JKR-BMS may now seem to be obsolete. However, viewed as a system, it is still in its infancy and more development work is needed to enhance and extend its usage to the entire JKR administration, that is down to the JKR District level.

The context of JKR-BMS is still as valid now as when it was first conceived. Thus the JKR-BMS still needs to satisfy similar functions namely:

(a) Provide information on bridges in the JKR road network

(b) Provide information on past and current bridge projects

(c) Allocate priority ranking for action

(d) Allocate budget and cost

(e) Provide alternative improvement decision

(f) Control of passage of abnormal vehicle

Thus the database, procedures and the analytical processes are still similar, though some aspects need refinement and modification. It is with respect to the user interface that can be much improved, to take advantage of the recent developments in IT, both in hardware and software. Some aspects of these proposed enhancements are highlighted below.

Routine for Prediction of Remaining Service Life

The next logical step of the inspection work is to begin building database on the deterioration nature, cause and extend. Once this is established the prediction of the remaining service life can be carried out using the latest research findings by experts in this challenging field. This can then be incorporated into the Improvement Proposal Module where data on service life is required for economic cost calculations.

Graphical User Interface

The JKR-BMS was written using DBASE IV under DOS, to run on IBM-Compatible PC-AT. The programming does provide drop-down menus, but these are basically single steps operation and do not provide for multiple opening of windows. This shortcoming can now be overcome with Windows' base operating system, such as Windows 95. The graphical platform not only allows easy and faster operations but open up other possible enhancements such as picture database and GIS.

Picture and Graphic Database

The review by the World Bank BMS experts had proposed that a picture database should be incorporated into JKR-BMS as an enhancement. Attempt was made, quite successfully by using graphical software called PicturePower and capturing of images using video camera. This was however found to be quite cumbersome process and took up a lot of storage space. However with the recent development of a Bridge Assessment software BRASS, it can be seen that presenting pictorial and graphical data is quite a simple process with a Windows base software. With the introduction of digital cameras in the market, the process will be further simplified. With this system graphics can be included in reports and it is suggested that inventory cards be produced using the JKR-BMS.

Geographical Information System

Attempt was also made to develop a GIS for bridges in the JKR-BMS inventory, and a prototype was completed in 1993. This was on the Windows platform using MAPINFO software, and thus was not directly linked to the JKR-BMS. Besides the lack of integration between the two softwares, another major shortcomings are the lack of accurate digitised base map for the whole country and the need to accurately referenced the bridges. A paper presented earlier by Ng and She has highlighted this issues. GIS is particularly useful in visualising abnormal vehicle movement, and in displaying visually locations of bridges under various selected conditions.

Conclusion

JKR-BMS has so far proven to be an indispensable tool in the management of quite a large bridge stocks under JKR jurisdiction. However more work

is required in order to improve the decision making process in so far as predicting the service life of the bridge stock and allocating resources for scheduled preventive maintenance works are concerned. JKR current strategy of conducting periodical inspection work will ensure that sufficient data be collected towards perfecting that decision making process.

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This paper was presented at the 2nd Malaysian Road Conference, Kuala Lumpur, 10 – 13 June 1996 and was judged as the best paper by a Malaysian author.

CONSTRUCTION OF LARGE-SCALE UNDERSEA FOUNDATIONS FOR THE HONSHU-SHIKOKU BRIDGES

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Introduction

The Honshu-Shikoku Bridges are links between Honshu, Japan's main island, with the island of Shikoku across the Inland Sea. There are three routes: Kobe-Naruto, Kojima-Sakaide, and Onomichi-Imabari, as shown in Fig. 1. These routes include the long-span bridges listed in Table 1. Generally, where bridge spans are longer, there are more cases of bridge foundations being constructed in the water rather than on shore, since this offers cost effectiveness by reducing superstructure work. Naturally, in such cases, it is preferable to place the foundations where the seabed is shallow and geological features are good. However, foundation placement is subject to limiting conditions such as navigation channels, fisheries operations, and overall geometric alignment. Some of the Honshu-Shikoku Bridges exceed 1,000 m in span despite foundations placed in the sea. Given the enormous dead, live, wind, and earthquake loads they have to transfer from the superstructures to the bearing stratum, they are of necessity very large in scale.

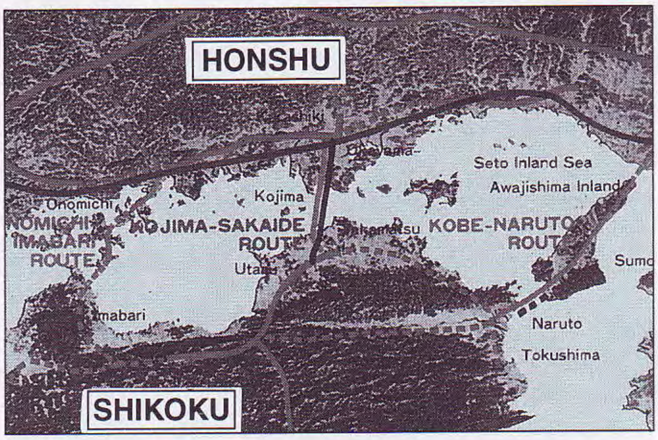

Fig. 1: Map of the Honshu-Shikoku Bridges

Table 1: Major long-span bridges in the Honshu-Shikoku Bridges Project

Route	Bridge	Bridge type	Center span (m)	Total Bridge Length (m)	Year completed	Number of undersea foundations
Kobe-Naruto	Akashi Kaikyo Bridge	Suspension	1,990	3,910	1998 (under construction)	2
	Ohnaruto Bridge	"	876	1,629	1985	3
Kojima-Sakaide	Shimotsui Seto Bridge	"	940	1,400	1988	0
	Kita-Bisan Seto Bridge	"	990	1,538	1988	3
	Minami-Bisan Seto Bridge	"	1,100	1,648	1988	3
Onomichi-Imabari	Innoshima Bridge	"	770	1,270	1983	0
	Tatara Bridge	Cable-stayed	890	1,480	1999 (under construction)	2
	Kurushima No.1 Bridge	Suspension	600	960	"	2
	Kurushima No.2 Bridge	"	1,020	1,515	"	2
	Kurushima No.3 Bridge	"	1,030	1,570	"	2

Table 2: Typical bridge foundations constructed by the laying-down caisson method

Bridge	Dimensions (m)	Depth below sea level (m)		Maximum tidal current (m/s)	Excavation volume (m^3)	Concrete volume (m^3)
		Seabed	Foundation			
Akashi Kaikyo Bridge 2P	ø 80 x 65	45	60	4	250,000	350,000
Bisan Seto Bridge 7A	75 x 59 x 55	25	50	1	600,000	240,000
Kurushima Bridge 9P	23 x 49 x 28	7	18	2	39,000	26,000

Undersea foundations are constructed using the same basic steps as used for land-based foundations: the subsoil is excavated, forms are installed, and concrete is placed. The primary requirement in the case of undersea work, where severe natural conditions such as strong tidal currents and waves influence the work, is quick implementation. As shown in Table 2, the major foundations for the Honshu-Shikoku bridges were constructed using the laying-down caisson method.

Laying-down caisson method

The laying-down caisson method of constructing underwater structures makes use of a caisson built in a dockyard; excavation of the seabed down to the supporting stratum and leveling in preparation for caisson placement are carried out in parallel with the caisson fabrication work, as shown in Fig.2. The caisson is towed and laid down on the seabed, and then filled with underwater concrete.

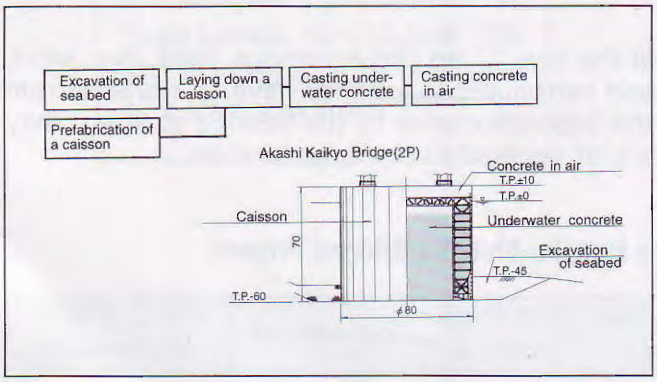

Fig. 2: Laying-down a caisson method

This method makes it possible to save excavation time by using large grab-bucket dredgers. Further, concrete can be cast immediately after excavation, since the caisson is prefabricated in a dockyard. The method can be considered a high-speed construction method which takes full advantage of buoyancy, one of the major advantages available in offshore work. Although the construction process is essentially very simple, work at depths of up to 60 m and in tidal currents with a velocity of over 4 m/s – as experienced in constructing the Honshu-Shikoku Bridges – requires very large construction equipment. For instance, grab-bucket dredgers with a bucket capacity of over 200 tf were used to excavate the seabed; winches with a maximum haulage force of 400 tf helped with laying down the caissons; and large floating concrete mixing plants are used to cast over 200,000 m^3 of concrete per caisson. Besides the large scale of the equipment, advanced construction management techniques are needed to meet the stringent requirements: the excavated seabed must be leveled to ±50cm, the caisson needs to be placed with an accuracy of ±1 m, and the compressive strength of the underwater concrete be not less than 180 kgf/cm^2.

The laying-down caisson method has been used to construct foundations for the Seto-ohashi Bridge, the Akashi Kaikyo Bridge, and the Kurushima Bridge in that order. This paper details the history of improvements made so far based on experience gained during the construction work; these are listed in Table 3.

Table 3: Progress of laying-down caisson method

Bridge	Year the foundations were completed	Excavation of Seabed	Laying-down caisson	Place underwater concrete
Seto-ohashi Bridge	1984	Underwater blasting → Excavate with grab → Large-diameter drilling	Mooring rope and floating crane	Prepacked concrete
Akasi kaikyo Bridge	1991	Excavate with grab	Mooring rope	Antiwashout underwater concrete
Kurushima Bridge	1994	Crashing of large rocks → Excavate with grab	Mooring rope	Antiwashout underwater concrete

The Seto-ohashi Bridge

(1) Excavation of the seabed

At the site of the Seto Bridge (see Photo 1), which was completed in 1988, a total of eleven foundations were constructed. The underwater concrete volume was 560,000 m^3. To excavate large volumes of granite from the seabed down to the bearing stratum, undersea blasting was used. Debris was then removed using grab-bucket dredgers, and the seabed was leveled using a large-diameter drilling machines. As shown in Photo 2, holes were first drilled into the seabed at intervals of 2m to take the explosives; this was facilitated by drilling machines mounted on self-elevating platforms. The maximum amount of explosive used in one operation was 3tf, with about 50 kgf explosives charged into each hole. Blasting debris was removed using grab-bucket dredgers with 125-tf buckets. The remaining projections and depressions on the seabed were leveled to around ± 50 cm using 2.5 m-diameter platform-mounted drilling machines.

Photo 1: The completed Kojima-Sakaide Route (Minami-Bisan and Kita-Bisan Seto Bridges)

(2) Laying down caissons

The largest of the steel caissons is 75 x 59 m in plane and is 55 m high. Its total weight is 18,000 tf. During towing, the space between the walls of the double-walled caisson was sealed. When it reached to the site, winches mounted on the caisson top were used to guide the caisson into the correct position by hauling on mooring ropes attached to eight preset sinkers. The caisson was maneuvered to 1 m above the seabed by pumping sea water into the wall space, and then a soft landing was achieved using floating cranes, as shown in Photo 3. The wall space was partitioned

Photo 2: Leveling the seabed using a drilling machine mounted on a self-elevating platform

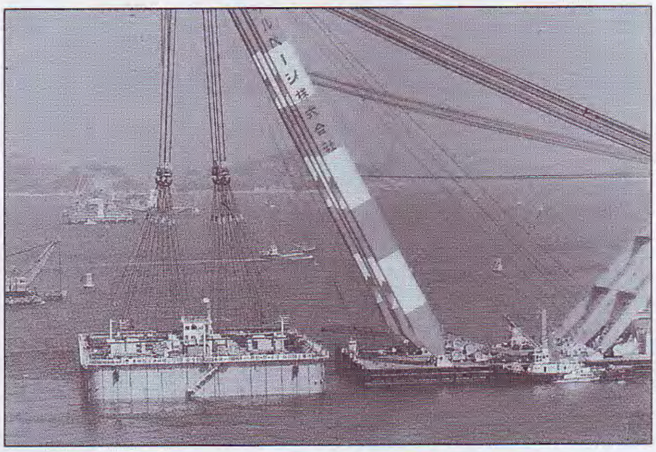

Photo 3: Caisson being laid down using a floating crane

into a number of sections to prevent a loss in stability as free sea water moved around. The position of the caisson was measured using two electro-optical distance meters; data was transmitted to computers on the caisson, where it was processed and the position displayed on CRTs. The winches delivered a haulage force of 130 tf using 76 mm-diameter cables. The caissons were laid down with an error of not more than 50 cm; the best accuracy achieved was 10 cm.

(3) Casting underwater concrete

Underwater concrete was cast by the prepacked concreting method: that is, the caisson was filled with coarse aggregate 80-150 mm in diameter, and special mortar was poured into the voids between the aggregate to form a monolithic concrete block, as shown in Fig.3. The method cuts in half the time and effort entailed in mixing the concrete, since the void ratio of the coarse

aggregate is about 50%. It also offers the advantage that the concrete is resistant to washing out even when cast underwater. Since it is impossible to form joints in prepacked concrete underwater, the caisson was partitioned into sections such that about 30,000 m³ of concrete could be cast in one operation. Each section was filled with mortar over a period of three consecutive days. A special barge was fitted with three mortar mixing lines, each with a capacity of 120 m³/hr, as shown in Photo 4. With the two lines in continuous operation, it took four years to cast the total of 560,000 m³ of concrete, and the record amount placed in one day was 12,000 m³.

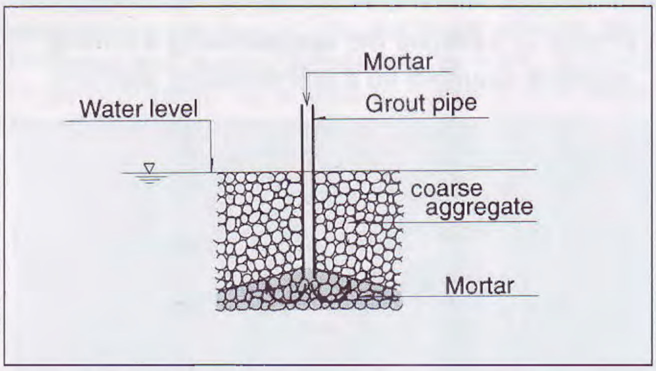

Fig. 3: Prepacked concrete method

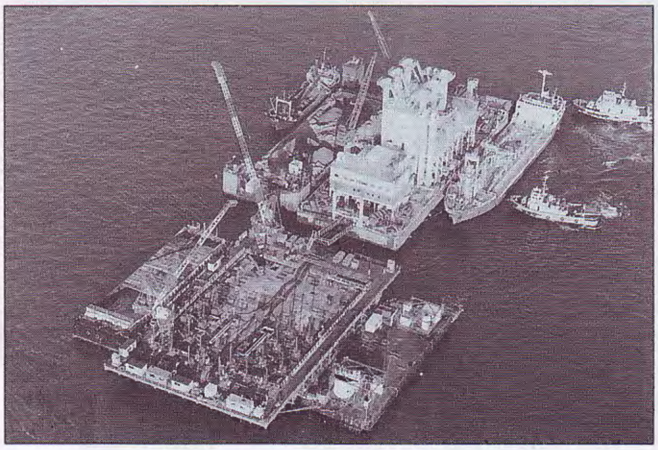

Photo 4: Concrete casting using a mortar mixing barge

(4) Casting concrete in air

The above-water sections of the foundations are of reinforced concrete structure, so as to provide a firm base for the anchor frames, which secure the bridge towers, and to place the steel reinforcement needed to withstand the tension acting on the concrete at the foundation top. For this work, each caisson was partitioned into 3 to 5 sections, and 700 to 1,000 m³ concrete was cast in one operation to achieve a lift of 0.5 to 1 m.

The Akashi Kaikyo Bridge

(1) Excavation of the seabed

The Akashi Kaikyo Bridge is still under construction, and (as of 1996) the stiffening girders are being erected, as shown in Photo 5. Both tower foundations were constructed underwater (with a concrete volume of 500,000 m³). The bearing stratum, which consists of the Akashi layer (sand gravel) and the Kobe layer (unconsolidated sandstone), was excavated in readiness for the foundations using grab-bucket dredgers. Large 250-tf buckets were used so as prevent a loss in excavation efficiency due to drifting of the bucket in the strong tides, which reach 4 m/s, as shown Photo 6. The maximum excavation depth was 60 m. To improve accuracy, topographic surveys were carefully repeated using ultrasonic bathymeters. In contrast with the Seto-ohashi Bridge foundations, the seabed was leveled without use of large drilling machines — reflecting the soft bearing stratum and improved surveying technology.

Photo 5: The Akashi Kaikyo Bridge: stiffening girders being erected

(2) Laying down caissons

Although the construction method adopted was basically the same as used for the Seto Bridge, cylindrical double-walled caissons were used to reduce tidal forces since the maximum tidal velocity around the foundations exceeds 4m/s. The caissons - which are 80 m in maximum diameter by 65m in height - were moored with eight cables, each capable of carrying a load of 400 tf. It is difficult to manually splice such mooring ropes, which were 120 mm in diameter, so the special quick-connecting joint shown in Photo 7 was developed. The mooring ropes were gripped securely with wedges and wound up using linear winches; the grippers were driven by hydraulic cylinders. The two caissons were landed softly on the seabed with an error of not more than 10 cm by allowing sea water into the caisson.

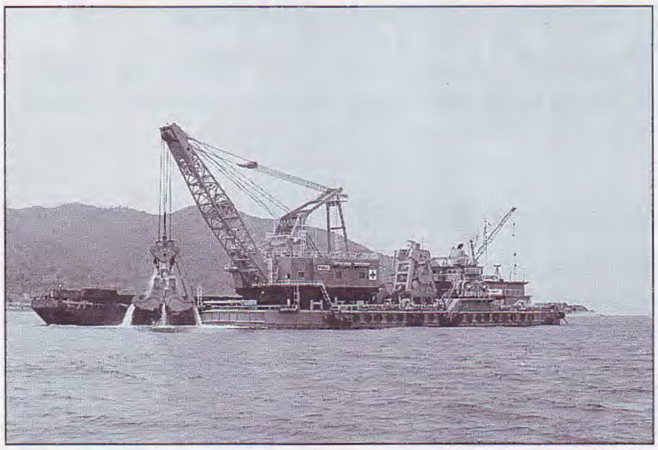

Photo 6: Excavation of seabed using a large grab-bucket dredger

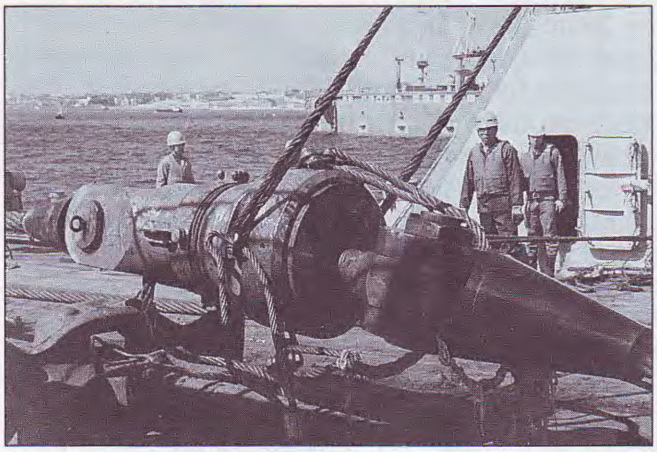

Photo 7: A quick-joint for splicing mooring ropes

(3) Casting underwater concrete

Instead of the prepacked concrete used in the case of the Seto-ohashi Bridge, antiwashout underwater concrete was placed in the caissons. This is concrete to which an antiwashout admixture and a high-performance water-reducing agent are added. Such concrete offers excellent resistance to washout as shown in Photo 8, and has good self-leveling characteristics as shown in Photo 9. Since the concrete also remains flowable over long distances and can be jointed underwater, the area within the 56 m-diameter inner wall was cast in a single lift of 3.5 m. Before proceeding with the subsequent lift, the concrete surface was cleaned using underwater cleaners. The space between the inner and outer walls was partitioned into 16 sections to prevent pumped sea water from flowing free, and casting was continuous to give a single lift of 50 m from bottom to top. Two concrete mixing lines with 3 m^3 mixers were mounted on a barge, as shown in Photo 10, for each foundation. Each caisson was cast in 30 operations, with 9,000 m^3 per operation. It took a year to cast the total of 500,000 m^3 of concrete for the caissons.

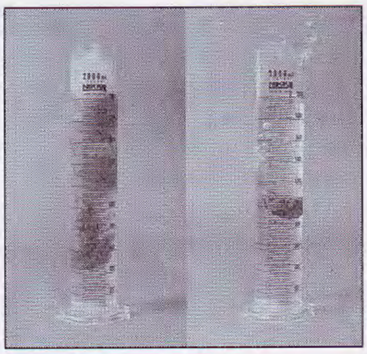

Photo 8: Antiwashout underwater concrete and ordinary concrete flowing in water

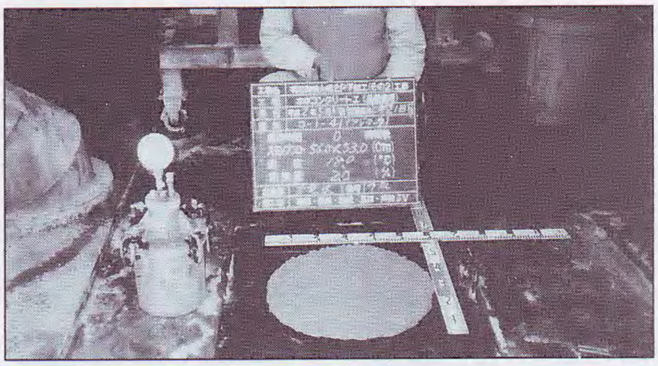

Photo 9: Self-leveling characteristics of antiwashout underwater concrete

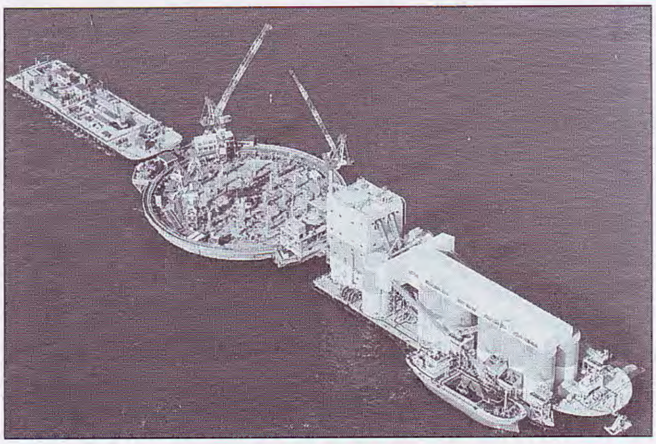

Photo 10: Casting underwater concrete using a concrete mixing barge

(4) Casting concrete in air

Upper part of the foundation is reinforced concrete structure and concreted in air. Each 80 m-diameter caisson was partitioned areawise into 7 sections, and each section was cast at a lift of 1 to 1.5 m using 700 to 1,500 m^3 of concrete in one operation.

The Kurushima Bridges

(1) Excavation of the seabed

At the site of the Kurushima Bridges, 5 foundations were constructed with a total underwater concrete volume of 120,000 m^3. To excavate the bearing

stratum for the foundations, which was granite as in the case of the Seto-ohashi Bridge, a drop chisel was used instead of undersea blasting. This entails dropping a chisel under gravity to break up the rock. As shown in Photo 11, a 50-tf drop chisel was used to break the rock to a depth of 1 m. The debris was then removed using grab buckets. As with the Akashi Kaikyo Bridge foundations, the seabed was leveled without the use of large drilling machines.

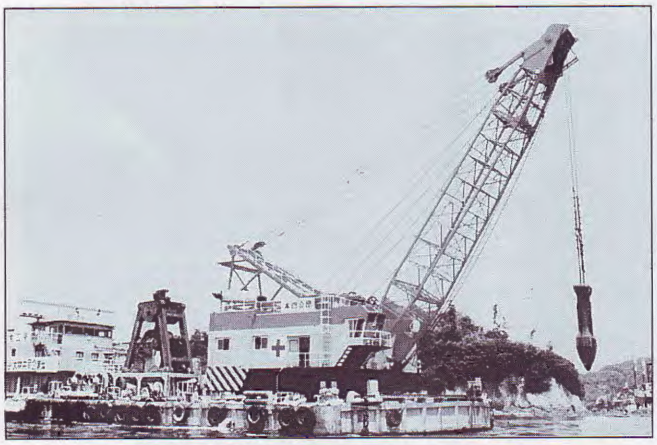

Photo 11: Breaking seabed using a drop chisel

(2) Laying down caissons

The foundations were basically constructed using the same method as for the Seto-ohashi Bridge, except that the caissons themselves are of concrete, as shown in Photo 12. This contrasts with the steel caissons used as forms for concrete placement in the case of the Seto-ohashi and Akashi Kaikyo Bridges. Concrete caisson was adopted at two pier which located in shallow water.

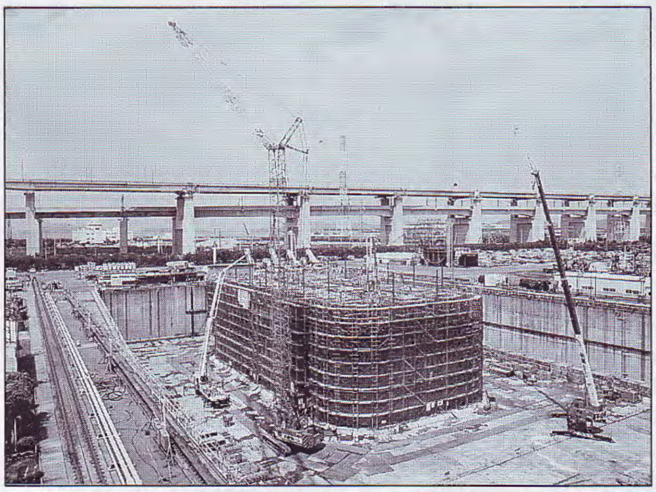

Photo 12: A concrete caisson being prefabricated in a dockyard

(3) Casting underwater concrete

Underwater concrete was cast using the same concrete barge as used for the Akashi Kaikyo Bridge.

(4) Casting concrete in air

Each caisson was divided areawise into two sections, and each was cast with a lift of 1 to 2 m at a rate of 700 to 1,500 m^3 per operation.

Conclusions

Of the three Honshu-Shikoku Bridge routes, the Kojima-Sakaide route is already completed and was opened in 1988. The Kobe-Naruto route and the Onomichi-Imabari route are now at the peak of superstructure construction, with opening scheduled for 1998 and 1999, respectively. The bridge foundations, many of which are in deep water where strong tidal current, were constructed without difficulty using the laying-down caisson method, thus overcoming the highest hurdle in the way of successful completion. This reflects continuous efforts to optimize construction methods by those involved in the projects, as well as advances in related technology and the fundamental excellence of the laying-down caisson method.

CORRELATION BETWEEN CBR AND MINI SPT VALUES

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Abstract

The field CBR of subgrade soil has been correlated with Mini SPT values under identical conditions of test. Tests were conducted for in-situ and saturated conditions of soil. The CBR tests were done at 25 cm depth as suggested by IS code and the mini SPT at 25 cm and 50 cm below the existing ground surface. The analysis of test results showed that a non linear relationship exists between CBR and Mini SPT values of soil subgrade. However, the correlation between the two parameters under soaked conditions of soil was found to be poor. It may be due to the difference in penetration level in two tests, more swelling of top layer of the soil than the lower layers and more susceptibility of CBR to surface moisture.

Introduction

The prime objective of a pavement design is to determine the thickness of different layers so that the pavement structure can bear the load without any distress. One of the most popular methods of flexible pavement design is based on CBR value of subgrade soil. CBR method of design is responsible for more than half of all the pavements now under construction throughout the world. CBR test, though simple, is time consuming, expensive and requires strict adherence to the test procedure to obtain the reliable and reproducible results.

Existing procedure of CBR test to design the flexible pavement considers soil strength up to a depth of 25 cm only. In actual practice soil layers up to 100 cm depth will have effect on pavement durability and strength. Any change in the soil structure below 25 cm will certainly affect the pavement performance. The factors with the strongest influence on values obtained in the CBR test are the texture of the soil, its water content, compacted density and confining pressure. The same material may have very different stress-strain behaviour, depending upon the circumstance in which it is loaded. The viscous characteristics of the soil play important role in pavement behaviour and are not easy to characterize in CBR test. Moreover, use of CBR design method is intimately related to the experience of the engineers who apply it. The personal experience which is really the most important factor is very hard to extrapolate. Further, the method gives the total thickness requirement of pavement above a subgrade and this thickness value remains the same irrespective of the quality of materials used in component layers. The CBR test does not take into account the properties of subgrade material up to the significant depth. Therefore, time has arrived when the method based on the CBR test will have to be supplemented with some other tests.

Objectives

The study was undertaken with the following objectives:-

(i) To study the suitability of test such as Mini Standard Penetration Test (MSPT) in addition to existing CBR test in flexible pavement design.

(ii) To study the effect of moisture content on Mini SPT and CBR values of soil.

(iii) To study the role of Mini SPT in flexible pavement design considering the soil behaviour up to the depth of influence zone.

(iv) To develop a correlation between field CBR and Mini SPT values at different depths.

SPT and Mini SPT

Though a number of penetration tests have been developed for sub-soil investigation, yet it is the standard penetration test (SPT) which enjoys the maximum popularity and most of the soil investigations are not complete without conducting this test. In India the test has been standardized by ISI vide its code-2131 (1981). The test has found great application in all sub-soil exploration being done throughout the country. However, the test apparatus is heavy and involves lot of expenditure in initially procuring it, and then transporting to the site and conducting the test. Therefore, it was felt that utility of this test can be further extended to subgrade soil investigations if it is made more portable and easier to conduct tests with least expense. Keeping the principles of main SPT intact, Mohan, et al (1985) made modifications in the SPT apparatus to make it more handy. The new equipment is referred to as Mini SPT. The details of this new equipment have been discussed in a paper by Mohan, et al (1985). The salient features of SPT and Mini SPT are given in Table 1. The detailed sketch of the Mini SPT apparatus is shown in Figure 1. The total weight of all components in the modified equipment is approximately 60 kg. It is easy to transport and can be handled by two persons. The correlation between SPT and Mini SPT value as developed by Mohan (1983) is given in equation (1).

Penetration value of full SPT = $^1/_4$ (Penetration value of Mini SPT)(1)

Design of Experimental Program

The penetration resistance is empirically correlated with engineering properties of soil such as consistency, bearing capacity, angle of internal friction, cohesion etc. The CBR test is also a penetration test but the results cannot be used to evaluate the basic strength characteristics of soil due to the ad hoc nature of test. An extensive field testing program was therefore, undertaken to correlate the N values of Mini SPT with the CBR values of subgrade soil. The tests were conducted under field conditions which meant that the results represent what could be expected under field conditions rather than under ideal "laboratory type" conditions.

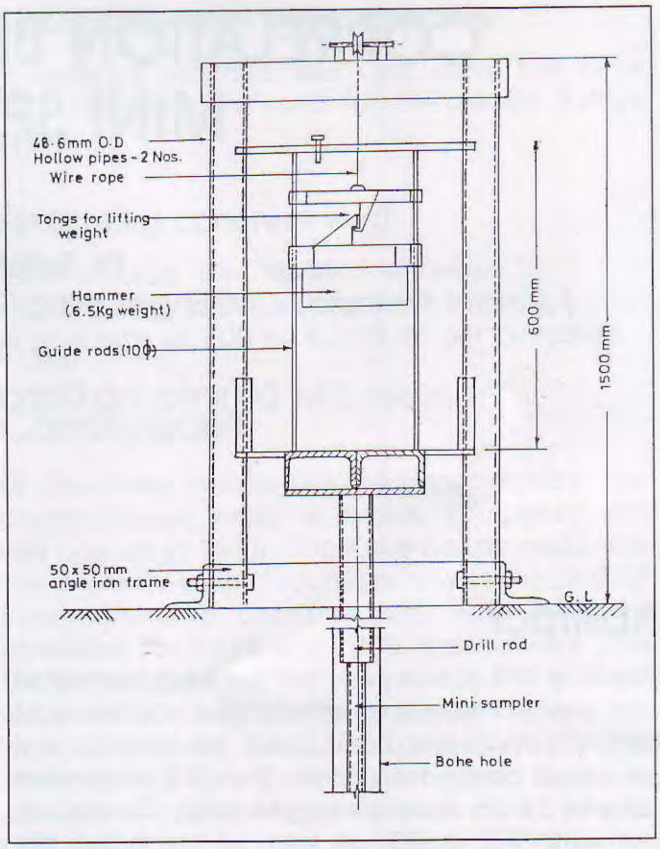

Fig. 1: Mini SPT Apparatus Along With Rod & Sampler *(not to scale)*

Table 1: Features of SPT and Mini SPT Apparatus

S1 No.	Dimension	Mini SPT	Main SPT
1.	Split Spoon Sampler		
	Outer diameter	26.0 mm	50.8 mm
	Inner diameter	21.0 mm	35.0 mm
	Thickness of cutting edge	2.0 mm	1.6 mm
	Length of cutting shoe	100.0 mm	75.0 mm
	Length of ball check value assembly	150.0 mm	150.0 mm
	Overall length	500.0 mm	675.0 mm
	Area ratio	53.28%	110.0%
	Metal used	Steel	Steel
2.	Hammer weight	6.5 kg	65.0 kg
3.	Height of fall	375.0 mm	750.0 mm
4.	Size of auger	75.0 mm	100.0 mm

Ten sites were chosen in Roorkee in and around the University campus at the distance of about 500 meters from each other to get a good range of soil properties. The locations of these sites are marked on a map of Roorkee shown in Figure 2. Two pits of 0.5 m x 0.5 m size and 25 cm deep below the existing ground surface were made. One of the pits was filled with water and left for 24 hours to saturate the soil. The field CBR tests were conducted in both the pits; one at in-situ condition of

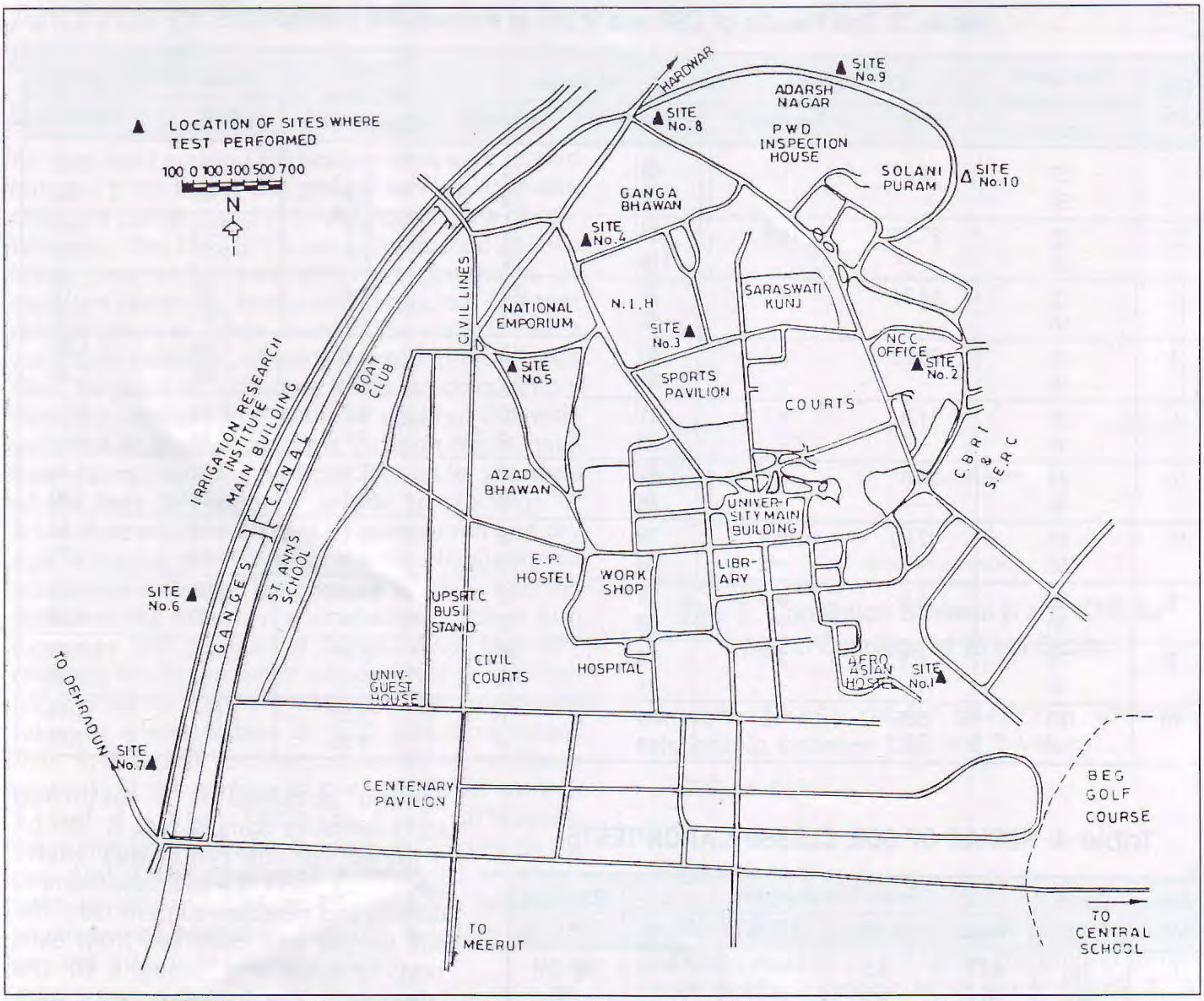

Fig. 2: Map of Roorkee Showing Location of Sites Wherte Tests Were Performed

moisture content and another at saturated condition of soil. As the eventual aim of this study was to observe the effect of soil properties at depth below 25 cm on pavement design, the Mini SPTs were conducted at two levels viz 25 cm and 50 cm below the ground surface. Like CBR tests, Mini SPTs were also conducted at in-situ and saturated conditions of moisture content.

Field CBR was determined using the method as described in IS: 2720 (Part XXXI) - 1969. The Mini Standard Penetration Test (MSPT) was conducted by means of split spoon sampler specified in Art. 3.0. The equipment provided a clean borehole 100 to 150 mm in diameter, for insertion of the sampler to ensure that the penetration test is performed on undisturbed soil. The bore holes were cleaned up to testing level using the auger with minimum mixing of soil from the bottom of the borehole. The sampler was lowered to the bottom of the bore hole and allowed to sink under its own weight. The sampler was then seated 5 cm with the blows of the hammer falling through 37.5 cm. It was further driven by 10 cm. The number of blows required to effect each 5 cm of penetration was recorded. The first 5 cm of drive was considered to be the seating drive and the total blows required for the next 10 cm penetration were termed as penetration resistance N.

The Mini SPT was conducted at each site in three bore holes close to each other and at two different levels viz 25 cm and 50 cm from ground surface The 'N' value of Mini SPT was taken as the number of blows required for the last 10 cm of penetration of the sampler after a seating drive of 5 cm. The results of field tests as obtained at different sites are given in Table 3.

Laboratory Tests

One of the advantages of Mini SPT is that along with N value of soil at a particular depth it gives the soil sample also at that level. Though the

Table 3: Test Results of CBR and MSPT at Various Test Locations

Site No.	Test Level from Ground (cm)	CBR Values		N Values		In-situ Moisture Content (%)	In-situ Density (gm/cc)	
		In-situ	Saturated	In-situ	Saturated		Bulk	Dry
1.	25 50	8.9 –	1.6 –	19 17	14 12	2.1 3.8	1..63 1.68	1.59 1.62
2.	25 50	27.73 –	13.5 –	97 116	29 30	8.3 7.1	2.03 2.01	1.87 1.81
3.	25 50	14.74 –	4.1 –	37 33	13 14	11.41 12.00	1.99 2.01	1.79 1.79
4.	25 50	26.76 –	3.6 –	40 30	15 12	6.26 7.46	1.58 1.59	1.49 1.49
5.	25 50	11.8 –	8.7 –	61 33	10 8	5.7 7.4	2.13 2.14	2.0 2.0
6.	25 50	30.0 –	6.8 –	34 28	17 14	16.3 15.4	2.02 1.84	1.74 1.60
7.	25 50	42.00 –	10.0 –	78 38	12 13	5.1 6.0	1.93 1.85	1.84 1.75
8.	25 50	5.5 –	4.5 –	18 18	13 12	14.0 10.0	1.90 1.63	1.66 1.48
9.	25 50	6.5 –	3.6 –	14 14	12 11	11.8 9.0	1.93 1.57	1.63 1.44
10.	25 50	5.9 –	4.5 –	10 9	5 4	5.67 8.96	1.57 1.57	1.49 1.44

Table 4: RESULT OF SOIL CLASSIFICATION TESTS

Site No.	Depth (cm)	Grain Size Analysis				Soil Type
		Sand	% Fine	L.L.%	P.L. %	
1.	25 50	93.5 96	6.5 4.0	– –	– –	SP-SM SP
2.	25 50	87.4 88	12.6 12	20.5 21.5	17.5 17.89	SM SM
3.	25 50	79.4 83	20.6 16	28.5 29.7	19.5 20.0	SC SC
4.	25 50	77.4 74	22.6 26	22.5 24.5	11.3 14.5	SC SC
5.	25 50	73.3 70	22.6 30	26.0 26.9	19.05 17.7	SC SC
6.	25 50	78.5 76	21.5 24	32 27	20.5 16.4	SC SC
7.	25 50	87 92	13 8	22 19.5	17.9 16.3	SM SP-SM
8.	25 50	60.3 59	39.7 41	29 31	25.34 24.13	SM SM
9.	25 50	89.5 87	10.5 13	22 21.5	18.2 17.5	SP-SM SM
10.	25 50	93.5 96	6.5 4	– –	– –	SP-S SP

SP = Poorly Graded Sand
SM = Silty Sand
SC = Clayey Sand

sample obtained is a disturbed representative sample, yet it can be utilized for carrying out many laboratory tests like soil classification, density and moisture content test. The soil sample was collected from each test location and brought to the laboratory. The following tests were performed on each sample in the laboratory.

- Soil classification test,
- Water content, and
- Bulk density and dry density tests

The soil classification was done in accordance with the method given in IS: 2720 (part IV) - 1975 and IS: 1498-1970. The results of grain size analysis and Atterberg limit tests on soils are given in Table 4. As may be seen, the type of soil encountered at various sites was of SM, SC & SP-SM type.

The moisture content of the soil taken from test location was determined as per the method given in IS 2720 (Part II) - 1973. The in-situ bulk density was also determined at each test location because CBR and SPT values are dependent on the density of the soil mass. These results are also given in Table 3.

Analysis of Results and Discussion

General

As discussed earlier, CBR tests in field were carried out at 25 cm below the ground surface at in-situ moisture content and saturated conditions of soil. Whereas, the Mini SPTs were conducted at two levels and under two different conditions of moisture contents, in-situ and saturated. The test results given in Table 3 are quite variable. Such variations however, should be expected between sites because of different levels of compaction, moisture content and material quality. The wide variation in N-value at some location might have been caused by large particles just under the shoe of the mini SPT sampler or due to blockage of drive shoe which is choked by compacted and dry soil. It maybe due to some drastic change in soil properties as well. This choking of shoe opening increases the effective area of shoe which in turn increases the number of blows. Also, Mini SPT readings are taken when sampler has penetrated approximately 150 mm whereas, CBR results are taken at a penetration of 12.5 mm. This means that in-site CBR is more susceptible to water present on the surface of the soil. Three sites 2, 5 and 7 (refer Table 3) have difference approximately half the N-value at 25 cm to 50 cm. And if the pavement is designed on these sites considering CBR at 25 cm only, failure of pavement is inevitable, because N-values at 25 cm and 50 cm depth vary widely.

Correlations Between CBR and N-Values

Based on impact and momentum laws, Harison (1986) derived the relationship between point resistance and penetration depth as given by equation (2).

$$R = \frac{W_1 h}{D} \cdot \frac{W_1 + e^2 W_2}{W_1 + W_2} \qquad (2)$$

where,
- R = Point resistance
- W_1 & W_2 = weight of hammer and instrument respectively.
- h = height of hammer fall
- D = penetration depth, and
- e = coefficient of restitution

The point resistance is an indication of strength of material under test. Therefore, it may be assumed that R can be represented as a function of such strength parameter as the CBR. Equation (2) indicates that R is inverse function of penetration depth, D (mm/blow), or direct function of number of blows (N) required for a specified penetration.

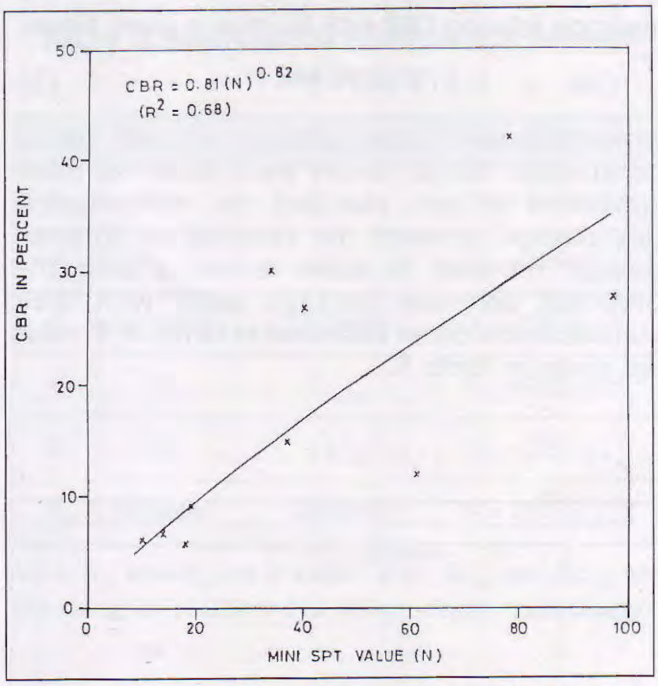

Fig. 3: Correlation Between N and CBR for In-situ Condition at 25 cm Depth

Harison (1986) finally gave the following relationship between CBR and D values.

$$CBR = A\,(D)^{-B} \qquad (3)$$

or

$$CBR = A\,(N)^{B} \qquad (4)$$

where A and B are constants. A plot between N and CBR values at 25 cm below the ground surface under in-situ condition is shown in Figure 3. As both CBR and MSPT are penetration tests and representative of soil strength, the trend shown in Figure 3 is quite expected. The mathematical

Table 5: Mathematical Relationships Between CBR and N Values

Sl No	Test Condition	Relationship	R^2 Value
1.	In-situ	$CBR = 0.82\,(N_{25})^{0.82}$	0.68
2.	Soaked	$CBR = 0.82\,(N_{25}) + 1.88$	0.41
3.	General	$CBR = 0.62\,(N_{25})^{0.87}$	0.61
4.	In-situ	$CBR = 1.02\,(N_{50})^{0.80}$	0.68
5.	Soaked	$CBR = 1.9\,(N_{50})^{0.40}$	0.12
6.	General	$CBR = 0.72\,(N_{50})^{0.86}$	0.55
7.	In-situ	$CBR = 0.24\,(N_{av})^{1.3}$	0.74
8.	Soaked	$CBR = 0.33\,(N_{av}) + 1.53$	0.33

NOTE: N_{25} and N_{50} are N value at 25 cm and 50 cm depth from existing ground surface and N_{av} is the average of N_{25} and N_{50}

equation relating CBR with N-value is given below.

$$CBR = 0.815 (N)^{0.822} \quad (5)$$

where N is the number of blows per 100 mm of penetration. Similar curves were drawn for other conditions of test also and the mathematical relationships between the variables as obtained through the best fit curve technique using the GRAPHER software package along with their statistical soundness measured in terms of R^2 value are given in Table 5.

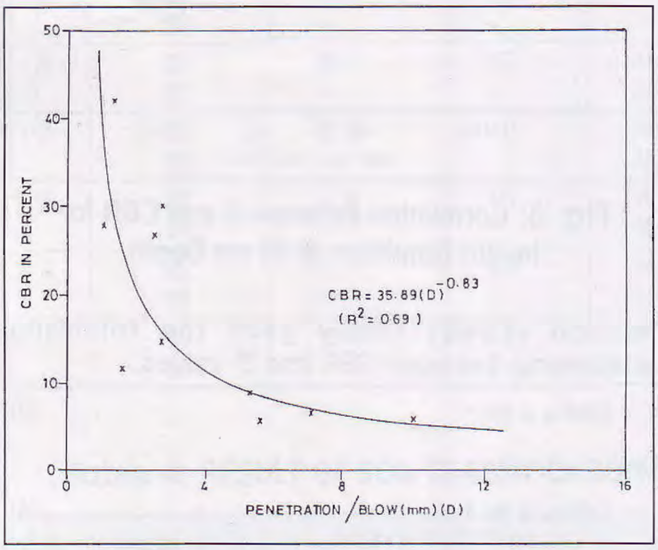

Fig. 4: Relationship Between CBR and D For In-situ Condition at 25 cm Depth

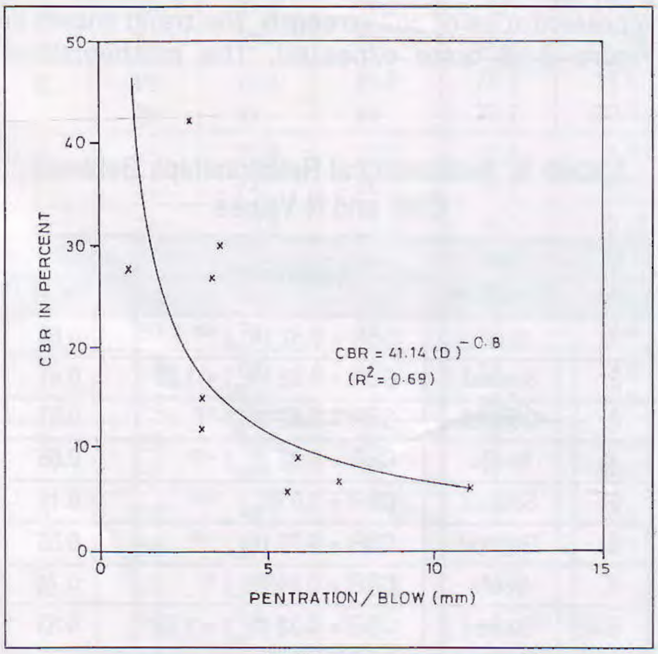

Fig. 5: Relationship Between CBR and D For In-situ Condition at 50 cm Depth

The plots between CBR and D value (Penetration per blow) were also made and are shown in Figures 4 and 5 for two conditions of test. The mathematical relationships are as given below.

$$CBR = 35.9 (D_{25})^{0.83} \quad (6)$$

and

$$CBR = 41.14 (D_{50})^{0.80} \quad (7)$$

where D_{25} and D_{50} are penetrations in mm/blow at 25 and 50 cm below the ground surface respectively. CBR in above equations is as determined at 25 cm under in-situ condition.

The results given in Table 5 indicate that the correlation between CBR and N-value under soaked condition of soil have low R^2 value. This may be due to the swelling of top layer of the soil and the fact that CBR test results are most affected by the moisture on surface. It may also be due to change in soil properties at depth below 25 cm. The difference in degree of saturation of soil layers might have also caused poor correlation between CBR and N-value. The combined correlation for saturated and in-situ test results, however, indicates a good relationship between the two parameters. A good relationship is also obtained between CBR at 25 cm and average of N-values at 25 cm and 50 cm depths and is given at serial 7 in Table 5. It may be mentioned here that the average value of N considers only those values of N_{25} and N_{50} which are close to each other.

Correlation Between CBR and Density

The CBR is not a constant characteristics of a soil but a circumstantial one, which depends on the water content and the compaction condition. There is a maximum CBR corresponding to a near optimum compacting water content in the laboratory. In present case CBR tests were conducted on in-situ conditions and hence, there was no control on water content and compaction. Analysis of test results shows that CBR is more dependent on dry density than moisture content. The dry density of soil is a measure of compaction. More is the dry density, greater will be the compaction of soil and greater will be the CBR value. The same trend is shown in the curve drawn in Figure 6. The mathematical equation relating CBR with dry density of the soil is given below.

$$CBR = 3 \times 10^{-4} (\gamma_d)^{20} \quad (8)$$

where γ_d is dry density of soil in gm/cc.

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